CHAPTER 13

INTERCHANGES

INTRODUCTION

The ability to accommodate high volumes of traffic safely and efficiently through intersections depends largely on what arrangement is provided for handling intersecting traffic. The greatest efficiency, safety, and capacity are attained when the intersecting through-traffic lanes are separated in grades. An interchange is a system on interconnecting roadways in conjunction with one or more grade separations that provides for the movement of traffic between two or more roadways or highways on different levels.

The type of interchange and its design are influenced by many factors, such as highway classification, interchange spacing, character and composition of traffic, design speed, and degree of access control. These controls plus signing requirements, economics, terrain, and right-of-way are of great importance in designing facilities with adequate capacity to safely accommodate the traffic demands.

Each interchange site should be studied and alternate designs made to determine the most fitting arrangement of structures and ramps and accommodation of bicycle and pedestrian traffic through the interchange area. Interchanges vary from single ramps connecting local streets to complex layouts involving two or more highways.

There are numerous combinations and types of interchanges. Refer to the current AASHTO publication *A Policy on Geometric Design of Highways and Streets* for many designs, like two-lane loop, ramp metering or Diverging Diamond, that may not be mentioned in this chapter.

Eight basic types of interchanges, labeled A through H, are shown in Figure 13-1:

- A and B are typical three-leg interchanges. A is a Trumpet interchange, named for the trumpet or jug-handle ramp configuration; B is a three-level, directional, three-leg interchange.
- C is an interchange with ramps in one quadrant that is not suitable for freeway systems but becomes very practical for an interchange between a major highway and a parkway. This design is appropriate for parkways because design speeds are usually low, large trucks are prohibited, and turning movements are light.
- D is a Diamond interchange. Diamond interchanges have numerous other configurations incorporating frontage roads and continuous collector or distributor roads.
- E is a Single Point Interchange.
- F is a Partial Cloverleaf interchange that contains two cloverleaf-type loops and two diagonal ramps. The configurations may vary to favor the heavier traffic movements.
- G is a Full Cloverleaf interchange, which gives each interchanging movement an independent ramp; however, it generates weaving maneuvers that must occur either in the area adjacent to the through lanes or on collector-distributor roads.
- H is a fully directional interchange or All Directional Four Leg Interchange.

Generally, the interchange type is predetermined during the scoping process, however final approval by the Federal Highway Administration is needed prior to conducting the Preliminary Inspection.
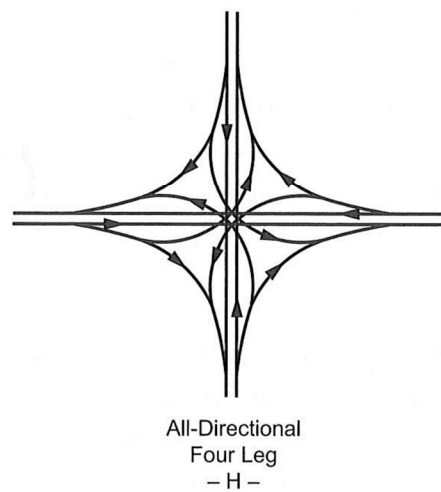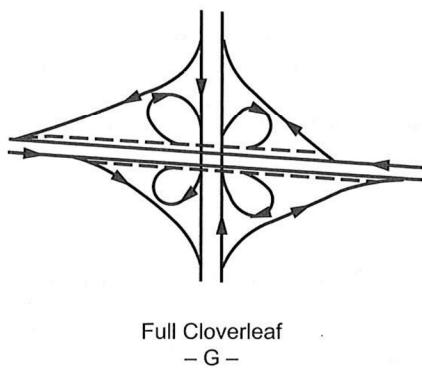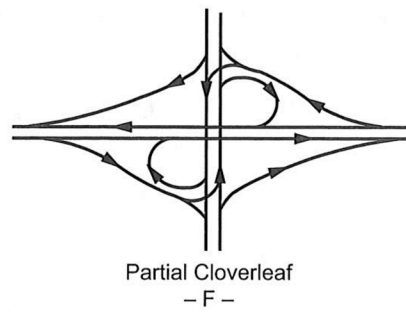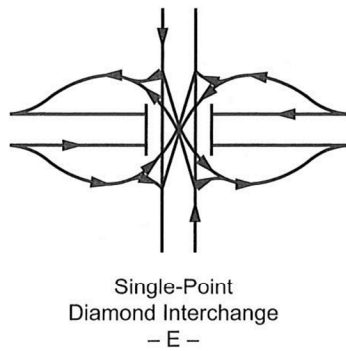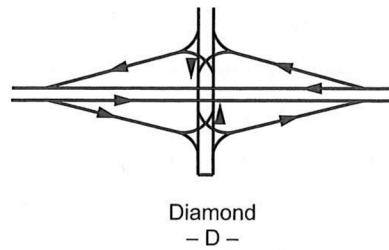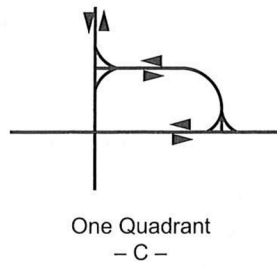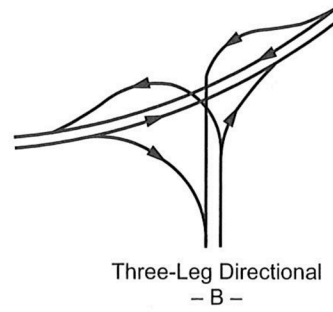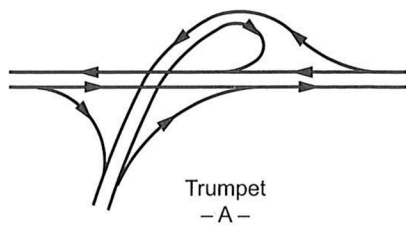
Figure 13-1 Interchange Types and Terminology

Ramp Naming Convention

Interchange ramps are to be lettered A through H. Ramp A will be the straight or outside ramp in the Northeast quadrant of the interchange. All other straight or outside ramps will be named in consecutive order in a clockwise pattern B, C, and D around the interchange. Ramp E will be the loop or inside ramp in the Northeast quadrant of the interchange. All other loop or inside ramps will be named in consecutive order in a clockwise pattern E, F, G, and H around the interchange. This naming convention shall be used regardless of the number of interchanges within the project scope.

```
                          N
                          |
        Ramps             |       Ramps
        D and H           |       A and E
                          |
 W ───────────────────────┼─────────────── E
                          |
        Ramps             |       Ramps
        C and G           |       B and F
                          |
                          S
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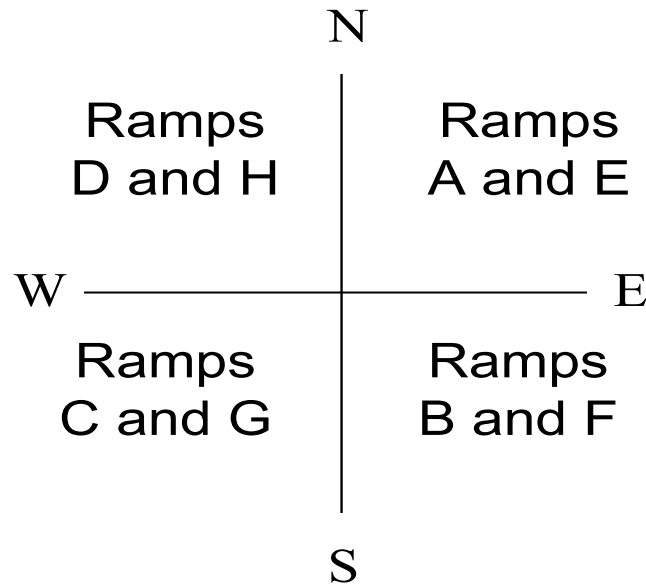
For Single Point Interchanges, the free right turn lanes are subordinate to the main (left turn) ramps. For this reason, the right turn lanes should be treated similar to loop ramps and be assigned the letters E through H beginning in the northeast quadrant and continuing clockwise.

During plans assembly, a layout of the entire interchange should be provided in addition to the usual plan sheets to minimize confusion regarding the interchange layout and ramp nomenclature.
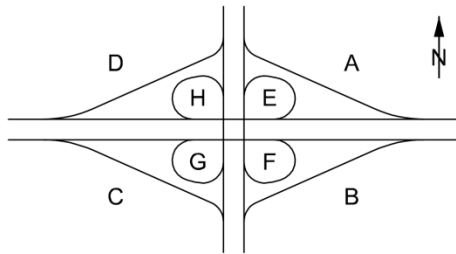
Full Cloverleaf
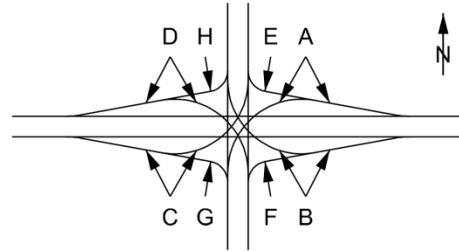
Single Point Interchange
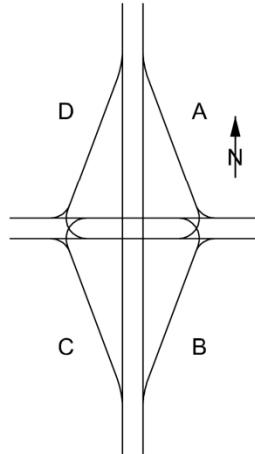
Diamond Interchange
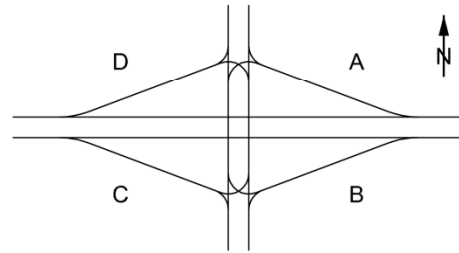
Diamond Interchange
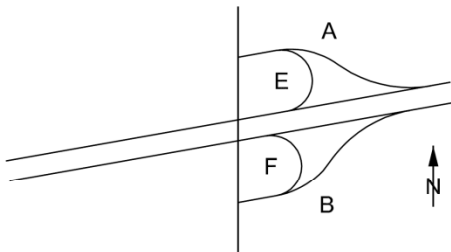
Partial Cloverleaf
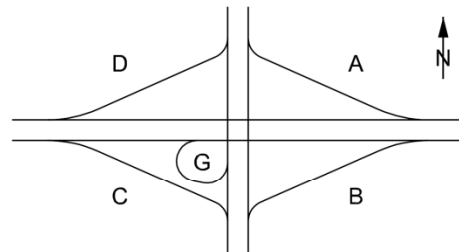
Additional Ramp to Diamond
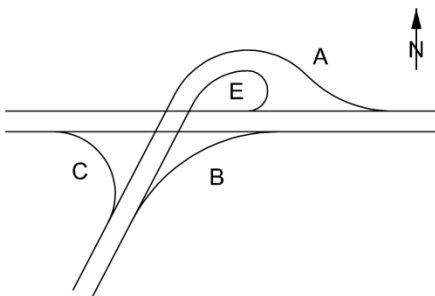
Trumpet
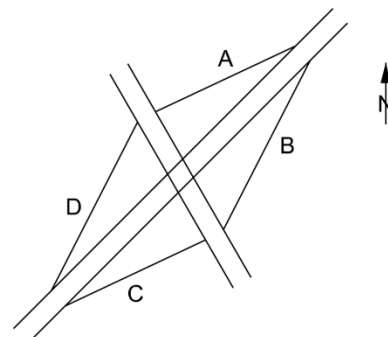
Skewed Interchange

Figure 13-2 Examples of Ramp Nomenclature

DESIGN OF AN INTERCHANGE

Interstate projects and intersections occurring at certain high volume locations on other systems are designed with grade separation structures to minimize traffic conflicts.

The standard design procedure presented in this chapter applies to the diamond type interchange with an approximately perpendicular crossroad, which is the predominant type of interchange used. Additional information is also provided for the design of a Single Point Interchange.

Figure 13-4 is a layout showing one half of the typical rural diamond interchange. The Parallel Entrance and Exit Ramp design is preferred. Figures 13-4a and 13-4b provides detail for the shoulder transition on both types of ramps.

The design of interchanges must be coordinated with the design of the mainline. The design process consists of the following:

- a preliminary design to set the locations of the ramps
- a calculation of ramp and intersection capacity
- a check of the design geometrics to ensure adequate sight distances
- a preliminary design inspection to make sure all concepts are feasible

Ramps

Interchange ramps are shown on the plan sheets by their alignments. This line is located on the inside edge of the ramp driving lane (Figure 13-3). As with mainline alignments the stationing of ramps should be laid out south to north and west to east.

Resurfacing projects shall follow the underlying plans for ramp alignments.

The following are design criteria associated with the standard diamond interchange used in South Dakota:

- 50 mph design speed R_{min} = 2300 ft
- Superelevation Rate (e) = 0.04 (e_{max} = 0.06)
- Typical Section as shown by Figure 13-3
  (single lane configuration/25' Ramp 2'-15'-8')

The minimum width for ramps with dual lanes is 28' (2-12' driving lanes with 2' shoulders) for slower speeds near ramp terminals. Directional ramps with 2 lanes on higher speeds (over 40 mph) the outside shoulder should be a minimum of 8'.

Once the ramp alignments have been established, Autoturn or other similar truck turning analysis using a WB-67 design vehicle must be completed to verify the ramp width needed. In addition, check the criteria in Chapter 12 – Intersections, Table 12-4 for the need for additional width. Case IIB is typically used, but other conditions should be reviewed based on operation and type of traffic expected at the interchange.
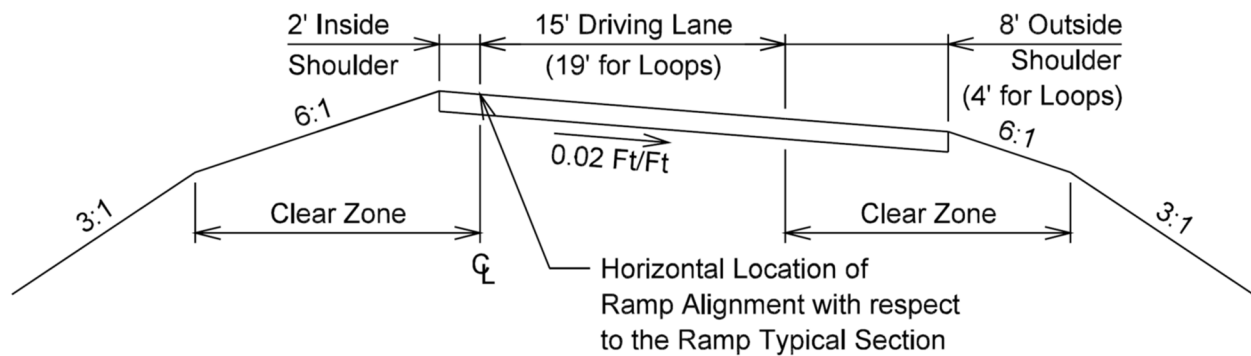
Figure 13-3 Ramp Typical Section

- Exit-ramp PI's are located 1200' from the centerline of the crossroad.

- Entrance-ramp PI's are located 1200' from the centerline of the crossroad.

- Intersection of a ramp with the crossroad is 550' from the centerline of mainline.

  The 550' distance, combined with the 1200' distance mentioned above, will allow for the ramp to intersect the cross road at a skew angle that does not hinder the horizontal sight distance or create a "looking over the shoulder" feeling. Truck turning analysis should be completed to determine the size of the radius needed on the inside angle of the ramp/cross road intersection.

  If mainline and cross road intersect at a skew, the designer should refer to Chapter 12 – Intersections to determine the need for squaring the intersection of the ramps with the cross road. If the intersection is squared, maintain the 550' distance along the cross road and ensure that the larger footprint of the ramp can be contained within available right-of-way or that additional right-of-way can be obtained.

- Exit-ramp with parallel type exit taper: The exit-ramp taper is a 20:1 taper for a distance of 240'. The minimum distance between the PC of the curve and where the 20:1 taper is 12' from the edge of the mainline outside driving lane line is 440'. This deceleration length is found in Table 13-2 for a 80 mph mainline speed and 50 mph ramp speed. However, longer parallel type deceleration lanes are more likely to be used properly by motorists than shorter lanes so a deceleration length of 800' is recommended. The lengths provided in Table 13-2 should be used as a minimum.

- Entrance-ramp with parallel type entrance taper: The minimum distance of the parallel acceleration lane between the PT of the curve and where the 50:1 taper is 12' from the mainline outside driving lane line is 980'. This acceleration length is found in Table 13-1 for a 80 mph mainline speed and 50 mph ramp speed. The entrance-ramp taper is then 50:1 for a distance of 600'.

The preceding minimum length requirements may be adjusted by using values in Tables 13-1 and 13-2. These values will need to be adjusted according to Table 13-4 if the ramp grades are greater than ± 2%.
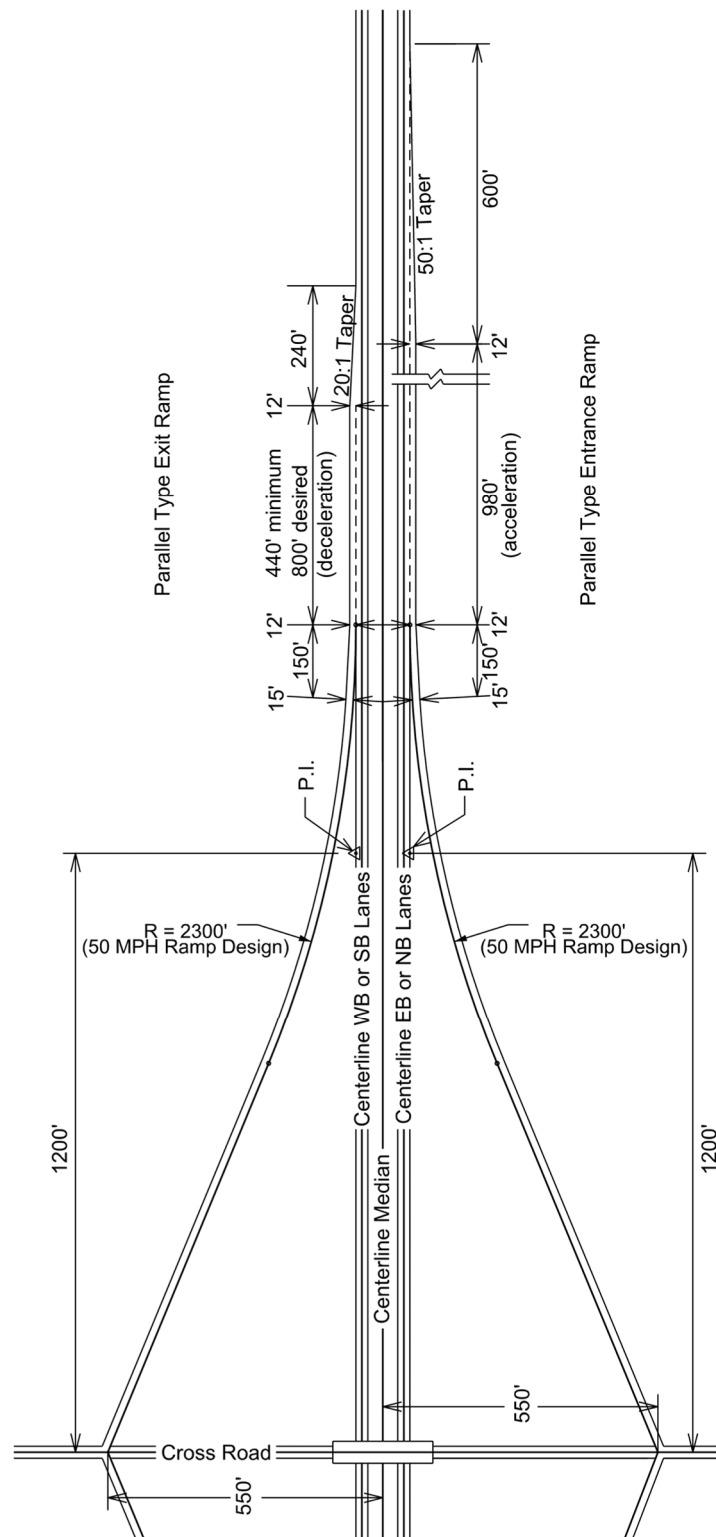
Figure 13-4 Typical Diamond Interchange

13-10
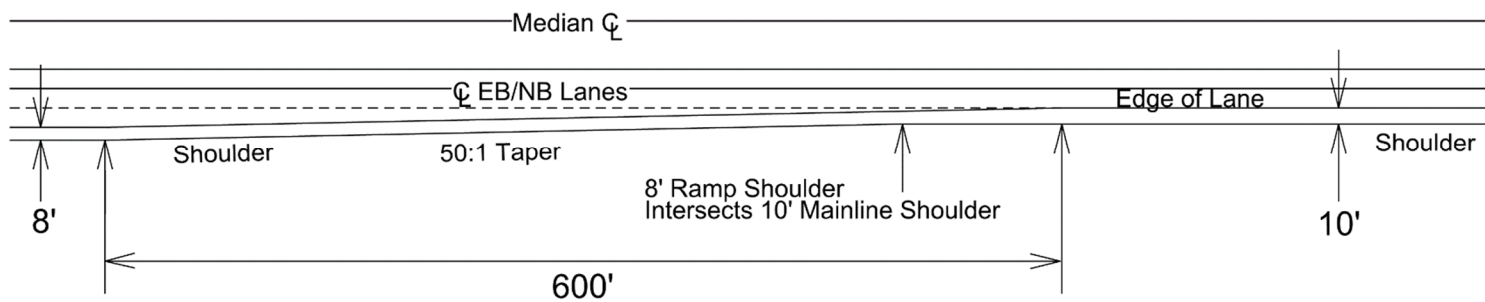
Figure 13-4a Shoulder Transition for Parallel Type Entrance Ramp
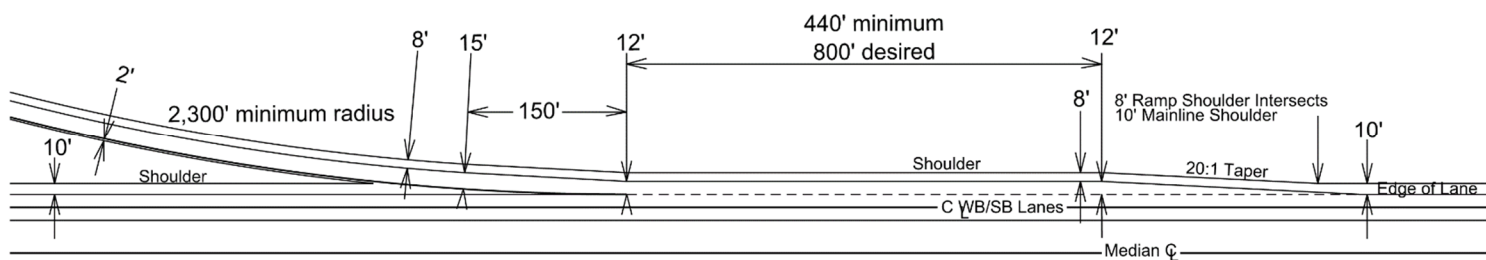
Figure 13-4b Shoulder Transition for Parallel Type Exit Ramp

Table 13-1 Minimum Acceleration Lengths for Entrance Terminals
With Flat Grades of 2 Percent or Less

Highway Design Speed (mph)	Speed Reached, V_a (mph)	Acceleration Length, L (ft) For Entrance Curve Design Speed (mph)								
		Stop Condition	15	20	25	30	35	40	45	50
		and Initial Speed, V'_a (mph)								
		0	14	18	22	26	30	36	40	44
30	23	180	140	--	--	--	--	--	--	--
35	27	280	220	160	--	--	--	--	--	--
40	31	360	300	270	210	120	--	--	--	--
45	35	560	490	440	380	280	160	--	--	--
50	39	720	660	610	550	450	350	130	--	--
55	43	960	900	810	780	670	550	320	150	--
60	47	1200	1140	1100	1020	910	800	550	420	180
65	50	1410	1350	1310	1220	1120	1000	770	600	370
70	53	1620	1560	1520	1420	1350	1230	1000	820	580
75	55	1790	1730	1630	1580	1510	1420	1160	1040	780
80	57	2000	1900	1800	1750	1680	1600	1340	1240	980

Note: Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration exceed 1300'.

Table 13-2 Minimum Deceleration Lengths for Exit Terminals
With Flat Grades of 2 Percent or Less

		Deceleration Length, L (ft) For Design Speed of Exit Curve, V'(mph)								
		Stop Condition	15	20	25	30	35	40	45	50
Highway Design Speed (mph)	Average Running Speed, V$_a$ (mph)	For Average Running Speed on Exit Curve, V'$_a$ (mph)								
		0	14	18	22	26	30	36	40	44
30	28	235	200	170	140	--	--	--	--	--
35	32	280	250	210	185	150	--	--	--	--
40	36	320	295	265	235	185	155	--	--	--
45	40	385	350	325	295	250	220	--	--	--
50	44	435	405	385	355	315	285	225	175	--
55	48	480	455	440	410	380	350	285	235	--
60	52	530	500	480	460	430	405	350	300	240
65	55	570	540	520	500	470	440	390	340	280
70	58	615	590	570	550	520	490	440	390	340
75	61	660	635	620	600	575	535	490	440	390
80	64	705	680	665	645	620	580	535	490	440
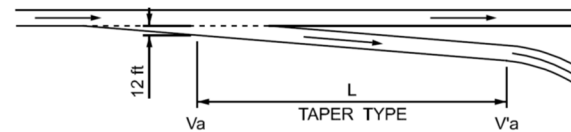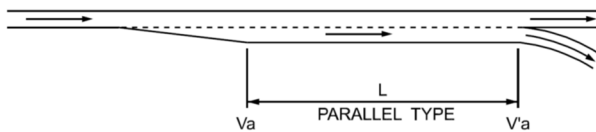
V = Design speed of highway   V$_a$ = Average running speed on highway

V' = Design speed of exit curve   V'$_a$ = Average running speed on exit curve

Table 13-3 Speed Change Lane Adjustment Factors as a Function of Grade

Acceleration Lanes

Design Speed Of Highway (mph)	Ratio of Length of Grade to Length of Level for Design Speed of Turning Roadway Curve (mph)[a]				
	20	30	40	50	All Speeds
	3 to 4 % upgrade				3 to 4 % downgrade
40	1.3	1.3	---	---	0.7
45	1.3	1.35	---	---	0.675
50	1.3	1.4	1.4	---	0.65
55	1.35	1.45	1.45	---	0.625
60	1.4	1.5	1.5	1.6	0.6
65	1.45	1.55	1.6	1.7	0.6
70	1.5	1.6	1.7	1.8	0.6
75	1.6	1.7	1.8	2.0	0.6
80	1.7	1.8	2.0	2.1	0.6
	5 to 6 % upgrade				5 to 6 % downgrade
40	1.5	1.5	---	---	0.6
45	1.5	1.6	---	---	0.575
50	1.5	1.7	1.9	---	0.55
55	1.6	1.8	2.05	---	0.525
60	1.7	1.9	2.2	2.5	0.5
65	1.85	2.05	2.4	2.75	0.5
70	2.0	2.2	2.6	3.0	0.5
75	2.15	2.35	2.8	3.25	0.5
80	2.3	2.5	3.0	3.5	0.5

Deceleration Lanes

Design Speed Of Highway (mph)	Ratio of Length of Grade to Length of Level for Design Speed of Turning Roadway Curve (mph)[a]	
All Speeds	3 to 4 % upgrade	3 to 4 % downgrade
	0.9	1.2
All Speeds	5 to 6 % upgrade	5 to 6 % downgrade
	0.8	1.35

[a]Ratio from table 13-3 multiplied by the length in Table 13-1 or Table 13-2 gives the length of speed change lane on grade.

If the mainline is on a curve, the ramp taper follows the mainline curvature so that the offsets from the pavement edge to the construction line are the same as the offsets on a tangent section. An example may be seen in Figure 13-5.

CA=C'A'   a=a'   b=b'   c=c'
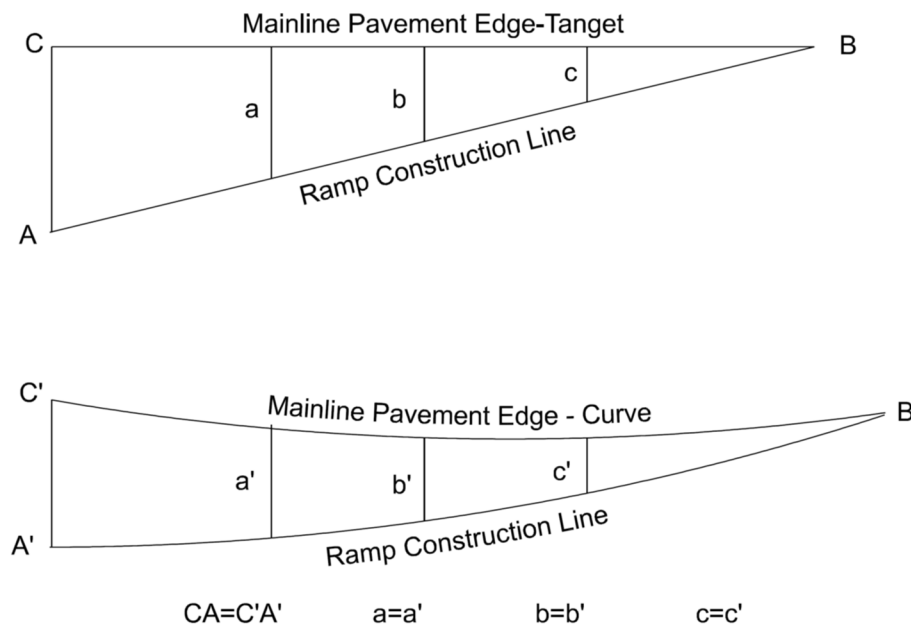
Figure 13- 5 Ramp Taper and Mainline Curvature

Loop Ramps

A design speed of 30 mph is preferred for loop ramps. A corresponding radius that meets or exceeds the design speed should be selected based on balancing the needs for traffic and impacts to the surrounding properties. Where right of way costs are lower (rural) a larger radius (300' for example) may be considered. Where right of way costs are high (urban) a smaller radius may be selected. If a lower design speed is considered, the design exception process as described in Chapter 2 - Scope Process shall be used.

- Superelevation Rate (e) = 0.06 (e_{max} = 0.06).

- Typical Section as shown by Figure 13-3 (single lane configurations)

Because of differences in traveling speeds for vehicles entering and exiting loop ramps in relation to mainline traffic speeds, the need for a deceleration lane (Table 13-2) entering a loop off ramp, and an acceleration lane (Table 13-1) exiting a loop on ramp should be considered on interchange resurfacing projects and included on all new loop ramp construction.

Where there is a loop to loop situation, such as at a full cloverleaf, or partial cloverleaf, where two loop ramps are adjacent to one direction of through lanes, collector-distributor roads (C-D roads) should be considered in the design of a new or reconstructed interchange. C-D roads transfer weaving movements from mainline to a separate roadway allowing a uniform pattern of exits and entrances to be maintained.

C-D road design speeds should not be less than 10 mph below the design speed of mainline, and outer separation between the C-D road and mainline should be as wide as practical, but at a minimum width to allow for proper shoulder widths and the installation of a suitable barrier, which will prevent indiscriminate crossovers.

Gore Areas

Figures 13-6 and 13-7 illustrate design criteria for gore areas.

The design control at the crossover line (not to be confused with the crown line normally provided at the centerline of a roadway) is the algebraic difference in cross slope rates of two adjacent lanes. Where both roadways slope down and away from the crossover crown line, the algebraic difference is the sum of their cross slope rates; where they slope in the same direction, it is the difference of their cross slope rates. A desirable maximum algebraic difference at a crossover crown line is 4 or 5 percent, but it may be as high as 8 percent at low speeds and where there are few trucks. The suggested maximum differences in cross slope rates at a crown line, related to the speed of turning traffic, are given in the Tables 13-4.

Table 13-4 Maximum algebraic difference in pavement cross slope
at turning roadway terminals

Design Speed of Exit or Entrance Curve (mph)	Maximum Algebraic Difference in Cross Slope at Crossover Line (%)
20 and under	5.0 to 8.0
25 and 30	5.0 to 6.0
35 and over	4.0 to 5.0

ENTRANCE RAMP GORE AREA

Parallel Acceleration Lane

8'
12'
2'
8'
15'
2'
10'
24'
4'

EXIT RAMP GORE AREA

Parallel Deceleration Lane

2' Paving Knockout

Gore Nose Detail
(Typically 20' Total Width)

10'
2'
8'

4'
24'
10'
2'
15'
8'
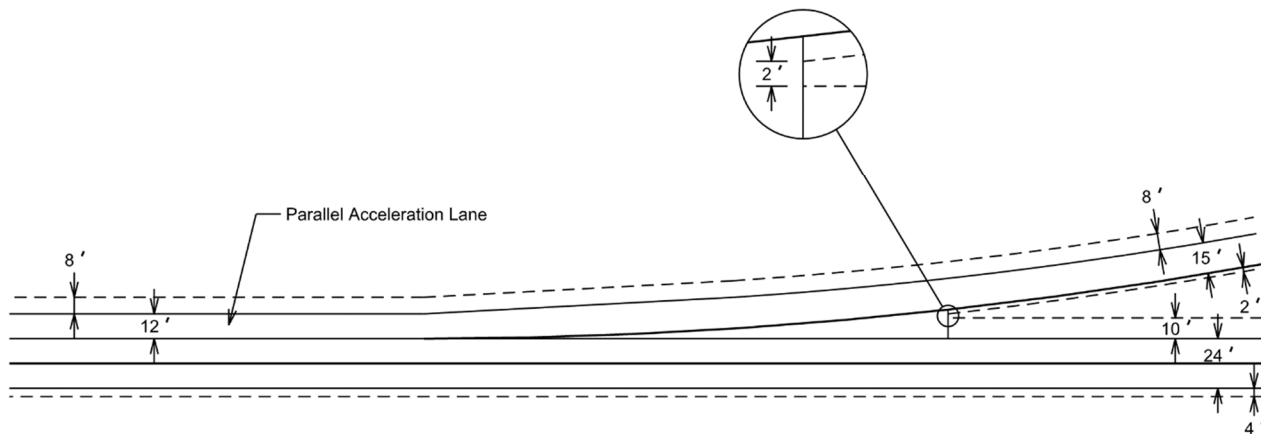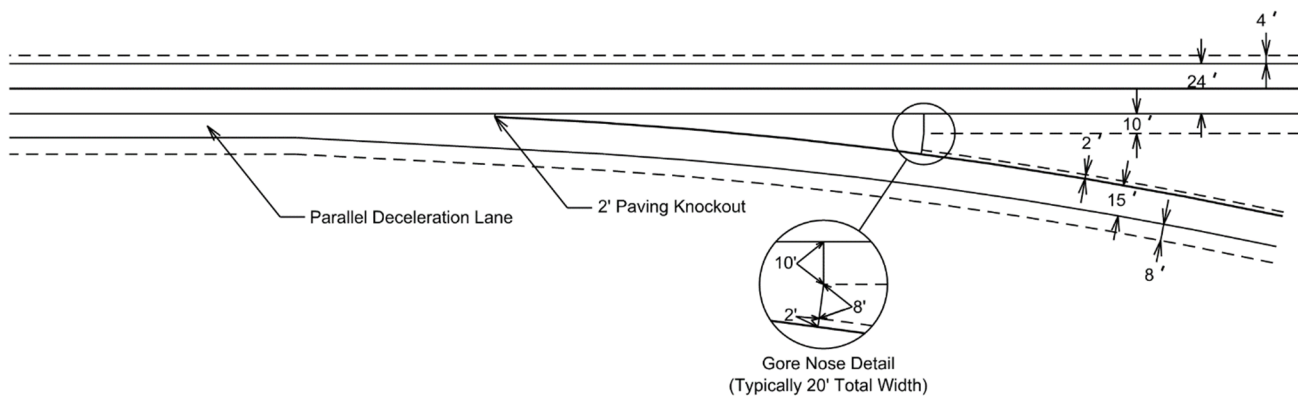
LEGEND

———————— Driving Edge

– – – – – – Shoulder Edge

Figure 13-6 Gore Detail

Apply the standard superlevation transition
to the ramp when mainline is superelevated.
Rollover maximums shall apply.

Direction
of Travel

Normal Crown

Direction
of Travel

Normal Crown

Direction
of Travel

When mainline is superelevated the
cross slope of the ramp shall match
the mainline cross slope

As the ramp grade approaches mainline grade the cross
slope of the nose area may vary from the ramp and mainline
cross sections, however rollover maximums shall apply.

ENTRANCE RAMP

As the ramp grade approaches mainline grade the cross
slope of the nose area may vary from the ramp and mainline
cross sections, however rollover maximums shall apply.

When mainline is superelevated the
cross slope of the ramp shall match
the mainline cross slope

Normal Crown

Direction
of Travel

Normal Crown

Direction
of Travel

Direction
of Travel

Apply the standard superlevation transition
to the ramp when mainline is superelevated.
Rollover maximums shall apply.

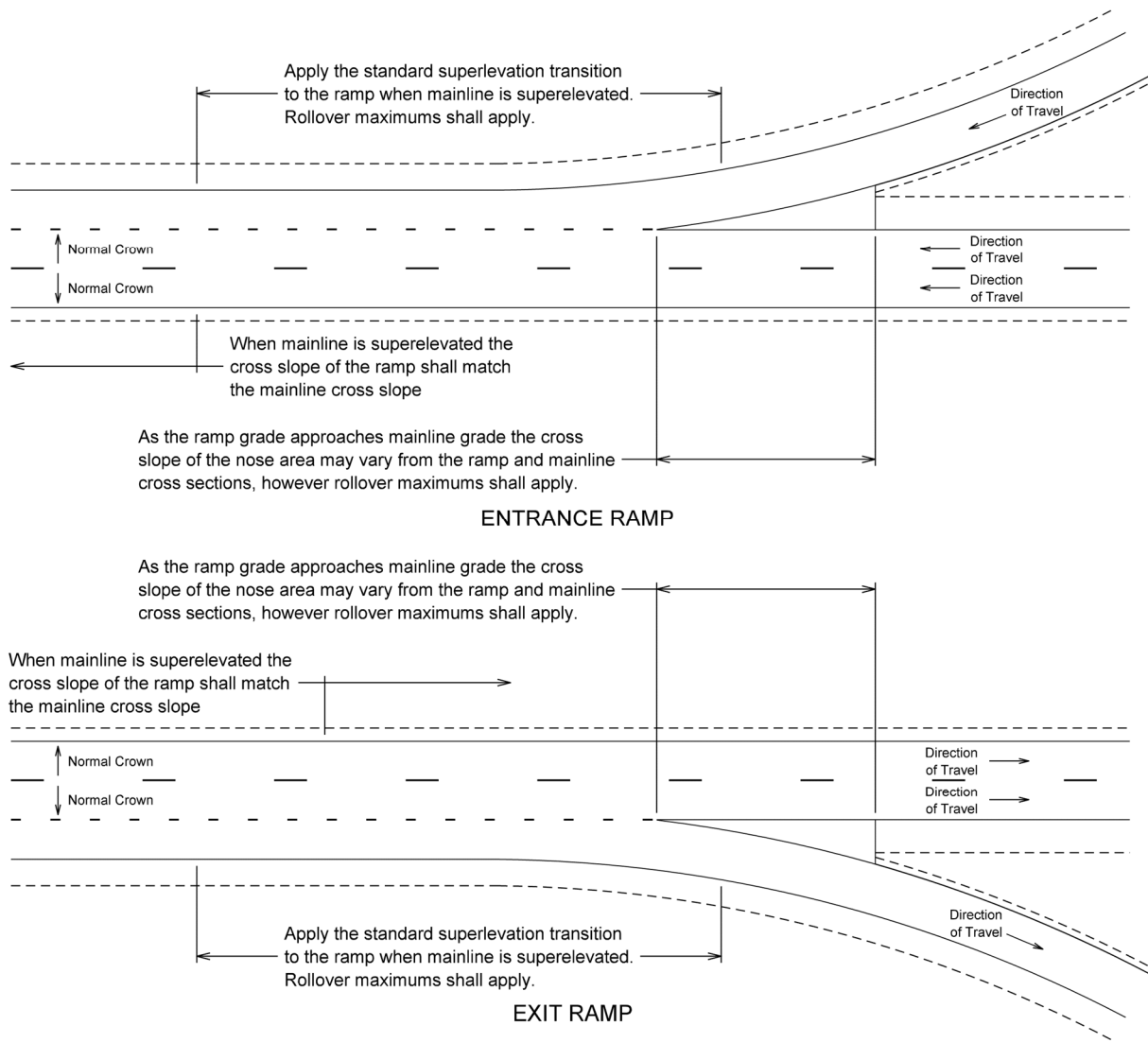EXIT RAMP

Figure 13-7 Cross Slopes in the Gore Area

Ramp Shoulders and Curbs

Shoulders should be provided on ramp terminals in interchange areas to provide a space that is clear of the traveled way to allow for emergency stopping, to minimize the effect of breakdowns on through traffic, and to aid drivers who may be confused.

The outside shoulder width as shown in Figure 13-3 will extend the entire length of the ramp, including the acceleration or deceleration lane to the point where the ramp shoulder intersects the mainline shoulder. Likewise the outside shoulder width as shown will extend around any returns or turn lanes at the cross road terminal end to match the cross road shoulder width.

When a mainline auxiliary lane between interchanges is utilized, the shoulder width of the auxiliary lane will be the same as the typical mainline shoulder width.

Except for Single Point Interchanges, ramps should be designed without curbs. Curb and Gutter should be considered only in the following instances:

- To facilitate particularly difficult drainage situations, such as in urban areas where restrictive right-of-way favors enclosed drainage.

- At the intersection of the ramp and the cross road, when the cross road has curb and gutter.

- At urban interchanges on the inside of the ramps adjacent to retaining walls.

In some cases, curb and gutter is used at the ramp terminals but omitted along the central ramp portions. Where curb and gutter is not used, full-depth paving should be provided on shoulders because of the frequent use of shoulders for turning movements.

On low-speed facilities, curb and gutter may be placed at the edge of the roadway. Vertical curb and gutter (Type B) is seldom used in conjunction with shoulders, except where pedestrian protection is needed. Where curb and gutter is used on high-speed facilities, sloping curb and gutter (Type F) should be placed at the outer edge of the shoulder.

There may be locations where curb and gutter is used on both sides of the ramps especially where there is restrictive right of way. In the event curb and gutter is used on both sides:

- One lane ramps: The curb and gutter on the left side can substitute the 2' shoulder. However, on the right side the curb and gutter would be in addition to the 8' shoulder.

- Multilane ramps: The curb and gutter can be placed on both sides without any shoulder (i.e. C&G – driving lane – driving lane – C&G) as this is typically at the ramp terminal transitioning to an urban cross road that does not have shoulders.

Designing Gradeline

The gradeline for the crossroad must be set with the mainline grade in mind. The crossroad grade must allow for minimum clearances. Typically the crossroad finished surface elevation would be 24' higher than the mainline finished surface elevation. The 24' elevation difference allows for the bridge deck thickness, girder depth and vehicle clearance under the crossroad structure. The 24' elevation difference is a starting point for design and is for the typical diamond interchange. A 17' vertical clearance should be attained between the bottom of the girders and the finished mainline surface (including future overlays). Vertical clearances must be coordinated with the Office of Bridge Design and the Office of Materials and Surfacing.

At a Single Point Interchange where the mainline crosses over, the traffic signals will be suspended under the mainline structure, within the girders. Exercise caution when designing the grades for exit-ramps so that the line of site to the signals is not impaired by steep grades. If the grade of the ramp cannot be adjusted for adequate line of site to the signals, work with the traffic signal designer to adjust the vertical location of the signals. Where the mainline crosses under, steep exit-ramp grades and/or steep crossroad grades will not necessary hinder the line of sight to the signals, but may cause the driver to have to look into the sun to observe the signal.

After the horizontal ramp alignments are laid out, gradelines for the ramps may be determined. Spline curves are often used to obtain acceptable ramp grades adjacent to the mainline grade and through the gore area.

Placing the horizontal alignment on the inside of the ramp lane will make it simpler to spline the vertical gradeline of the ramp to the mainline outside lane edge. This is particularly helpful when the mainline is in a superelevated curve.

For the exit-ramps, the cross slope of the crossroad should be extended onto the off-ramp a minimum distance of 100' to provide a relatively flat grade for stopped traffic to proceed from.

For the entrance-ramps, the ramp profile and cross slope needs to be analyzed against the cross road profile and adjusted accordingly to prevent trucks from overturning when making the left turn from the crossroad.

Ascending and descending grades on ramps should be 3% to 5% maximum. Ascending and descending grades on loop ramps should be 5% to 7% maximum.

Check Of Geometrics for Sight Distance Required

Available horizontal and vertical sight distances must be equal to or exceed the largest of the horizontal and vertical sight requirements.

After a general layout and preliminary gradelines have been set, vertical and horizontal sight distances must be checked. To check the horizontal and vertical sight requirements, complete these four steps:

- Check the "K" factors:

  Set vertical alignment for the crossroad checking to make sure the "K" factor meets minimum stopping distance for the required design speed. Discussion of the required "K" factors can be found in Chapter 6 of this manual.

  $$K = L/A$$

- Check horizontal sight distances for "P" vehicle:

  Make a check for Case B – "Intersections with Stop Control on the Minor Road" (Current AASHTO publication *A Policy on Geometric Design of Highways and Streets*). Case B1-"Left turns from the minor road" should be checked. Case B2 – "Right turns from the minor road" should be checked if no left turns from the minor roads are allowed.
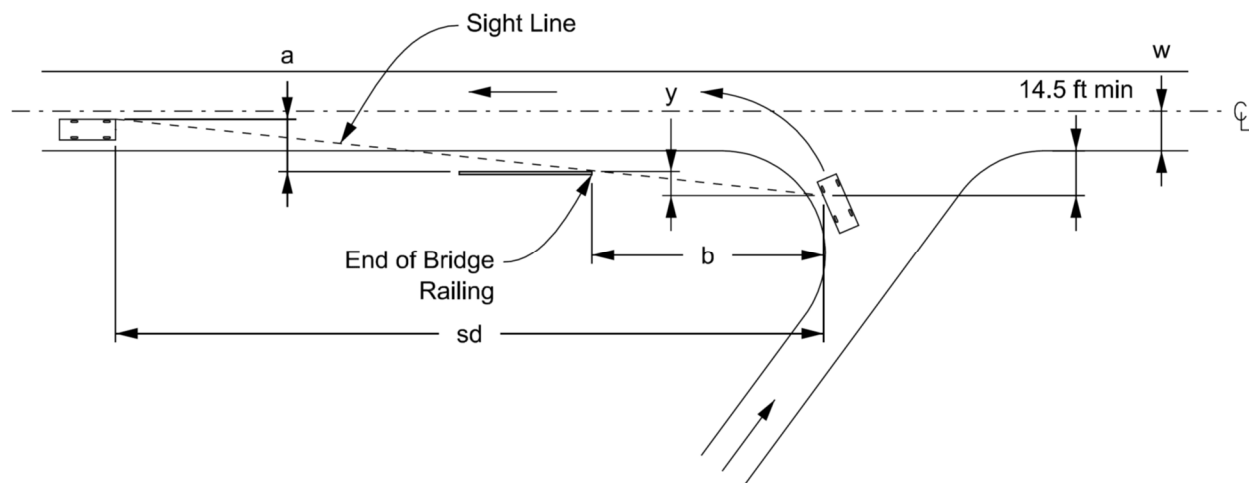
a = offset from driver's edge of through vehicle to edge of bridge railing = w + sw - 2.5'
y = offset from driver's eye to edge of bridge railing = 14.5' - sw
b = distance from driver's eye to the edge of bridge railing.

sd = sight distance = $\dfrac{b(a+y)}{y}$

sw = shoulder width
w = driving lane width

Figure 13-8 Measurement of Horizontal Sight
Distance at Ramp Terminals

- Find the actual horizontal sight distance from the equation on Figure 13-8. It will be necessary to estimate the new structure length and guardrail placements at this time. It may be helpful to create vertical and horizontal layouts of the crossroad and then determine the new line of sight.

  New interchange sight distances should be computed with the above described method. In locations where minimal available Right of Way or other restricting circumstances prevail, FHWA has approved a reduced offset distance for the car stopped at the exit-ramp. In these situations, FHWA must be contacted and will approve the reduced offset on a site-by-site basis. In the past, FHWA has approved using a dimension of 14' from the outside edge of the outside driving lane of the crossroad to the line of sight of the driver stopped on the exit-ramp (this offset still needs to be approved by FHWA for each site).

  Now compare the actual horizontal sight distance with the required sight distance for Case B1.

- Check stopping sight distance for on-coming vehicle:

  Use the following equation that applies

  (Current AASHTO publication Equation 3-42 *A Policy on Geometric Design of Highways and Streets*)

  When S is less than L, $L = \dfrac{AS^2}{100\left(\sqrt{2h_1}+\sqrt{2h_2}\right)^2}$

  (Current AASHTO publication Equation 3-43 *A Policy on Geometric Design of Highways and Streets*)

  When S is greater than L, $L = 2S - \dfrac{200\left(\sqrt{h_1}+\sqrt{h_2}\right)^2}{A}$

  L = length of vertical curve, ft.
  S = sight distance, ft.
  A = algebraic difference in grades, percent
  h_1 = height of eye aboveroadway surface, ft.
  h_2 = height of object above roadway surface, ft.

  For a "P" vehicle, height of eye = 3.5 ft
  Height of object = 2.0 ft

  Compare the available stopping sight distance with the required sight distance from the appropriate value from Table 13-5 or for the "P" design vehicle.

Table 13-5 Design Controls For Crest Vertical Curves
Based On Stopping Sight Distance

Design Speed (mph)	Stopping Sight Distance (ft)	Rate of Vertical Curvature, K (length (ft) per percent of A) Rounded for	
		Computed	Design
20	115	6.1	7
25	155	11.1	12
30	200	18.5	19
35	250	29.0	29
40	305	43.1	44
45	360	60.1	61
50	425	83.7	84
55	495	113.5	114
60	570	150.6	151
65	645	192.8	193
70	730	246.9	247
75	820	311.6	312
80	910	383.7	384

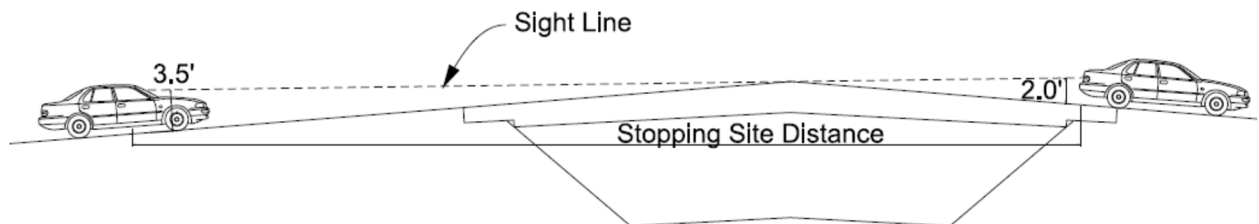**Figure 13-9 Stopping Site Distance**

Should one of the previous sight distance calculations fail for the selected speed, recalculate for Case B1 ("Intersections with Stop Control on the Minor Road, Left turns from the minor road" as found in the Current AASHTO publication *A Policy on Geometric Design of Highways and Streets*) – determining the design speed that the calculated sight distance corresponds to.

SINGLE POINT INTERCHANGE (SPI)

US16B & SD79 in Rapid City
Cross Road Over

I-29 & Madison Road in Sioux Falls
Cross Road Under

Figure 13-10 Examples of Single Point Interchanges in South Dakota

The design of a SPI will be an iterative process. It is suggested that the design of the SPI begin at the horizontal intersection of the mainline and the crossroad with the head to head clearance and then work outward to the gore areas, incorporating truck turning analysis as early as possible.

SPI's by the nature of their design, have opposing left turn movements sequenced together that pass at the right side of each other. Because of this unique movement, head to head vehicle clearance is critical. Optimally, an edge of lane to edge of lane clear distance of 10' should be obtained. Minimally, a clear distance of 4' is allowable.

The radius of the left turn should be in the range of 160 ft to 350 ft. Where right-of-way is not an issue, the radius may be made larger to reduce the lengths of retaining wall needed.

The radius of the right turn lanes should be in the range of 100 ft to 125 ft. Where space is limited, the right turn lanes can be treated like a side street intersection with the crossroad (perpendicular intersection), however a truck turning analysis needs to be completed to determine the size of the inside radius and the width needed.

SPI entrance-ramps and the corresponding left turn from the crossroad are typically designed as a dual left turn and two-lane entrance-ramp. The entrance-ramp continues on as a two-lane facility beyond the merge point of the right turn lane for a minimum distance of 600'. The transition to a single-lane facility is a minimum of 600' in length and must reach single lane width prior to or at the beginning of the ramp acceleration lane. A traffic analysis should be provided to determine if the minimum distances provide adequate time

for the merging of ramp traffic prior to the merging with interstate mainline traffic. At this point, entrance-ramp parallel entrance taper criteria govern.

Additional information on Single Point Interchanges can be found in NCHRP Report 345 Single Point Urban Interchange Design and Operations Analysis.

ADDITIONAL INTERCHANGE CRITERIA

In addition to the previously mentioned design criteria the following items must be considered.

Pavement Sections

Ramp vs. Interstate (mainline) – The pavement thicknesses must be verified with the Office of Materials and Surfacing. For ease of construction, the ramp pavement section adjacent to mainline is typically designed with the same pavement section as mainline. Once the ramp passes through the gore area, or starts to diverge from mainline, the recommended ramp pavement section begins. This difference in pavement thicknesses may necessitate a vertical change in the ramp gradeline (subgrade).

Match Lines

Cross section sheets are not wide enough to include the mainline, crossroad and ramp cross sections in one continuous section through an interchange area. The mainline cross sections are surveyed to include the ramp areas. The crossroad is cross sectioned separately with a skip through the mainline area. Therefore, it is necessary to develop separate ramp cross sections.

Each cross section in each area shares a common point with a cross section in the adjacent area. These common points fall on match lines. The designer, using engineering judgment and the limits of the survey, determines the location of the match lines. Figure 13-11 illustrates examples of utilizing match lines.

Ramp cross sections are perpendicular to the ramp construction line. Each ramp cross section is referenced to both the respective match line and the cross section on the other side of the match line. The separate cross sections for mainline, ramp, and crossroad areas are used to simplify earthwork computations.
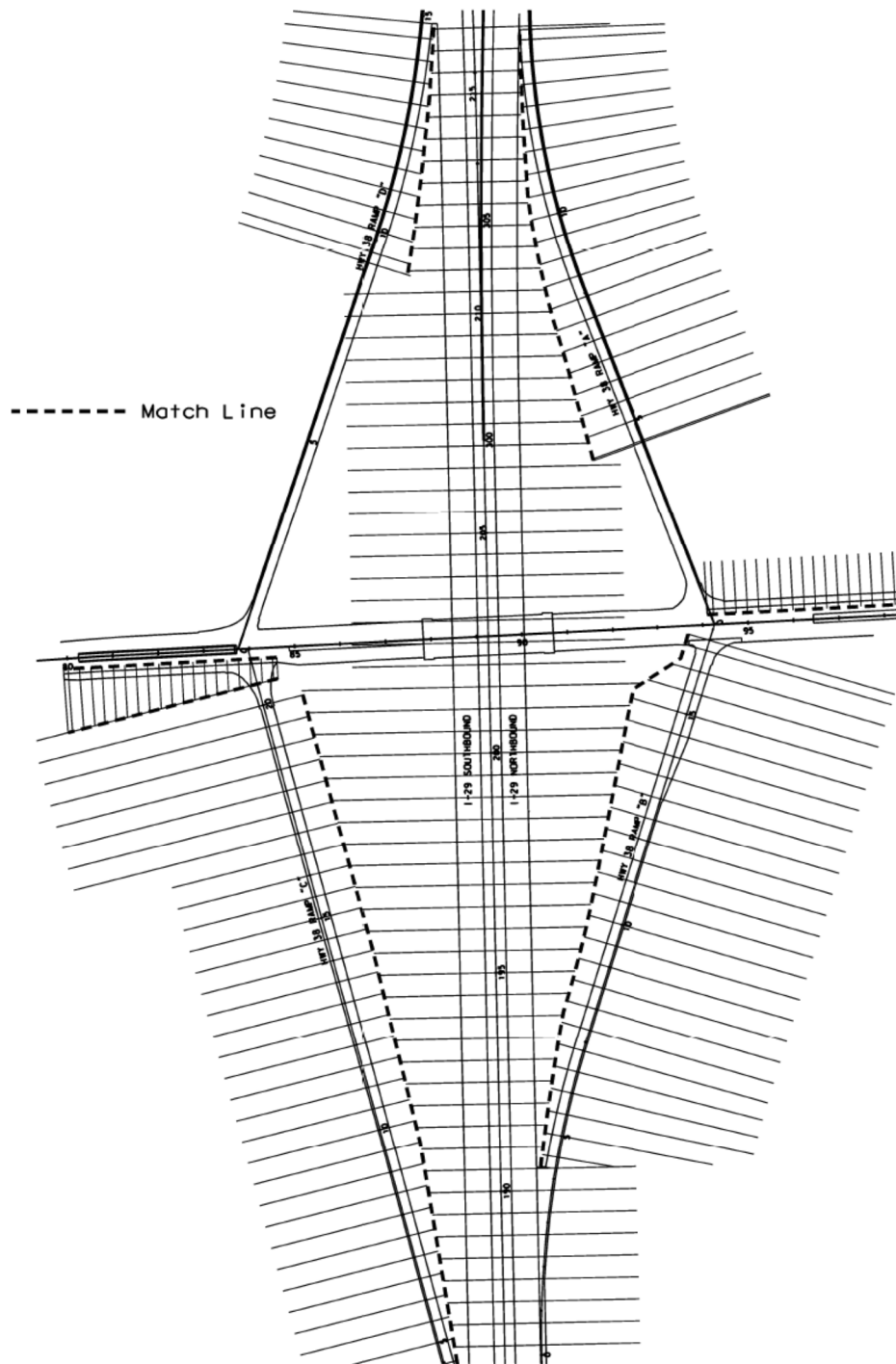
Figure 13-11 Match Line Layout

Control of Access

Control of access (COA) is established on the Interstate System at all interchanges and grade separation locations. The most significant design factor contributing to low crash frequencies for roadways is the provision of full control of access. Full access control reduces the number, frequency and variety of events of which drivers encounter. Therefore control of access can have a positive influence on the traffic flow and safety performance through the interchange area.

Establishment and acquisition of control of access should not be confused with access management (refer to Road Design Manual Chapter 17 – Access Management). Full access intersections and limited access intersections (right-in/right-out) may fall within the length of control of access as long as the minimum control of access from ramp terminal is provided per Table 13-6. The designer shall coordinate with the appropriate access management specialist and traffic engineer.

Control of access shall extend the full length of ramps and ramp terminals on the crossroad. Such control shall be either acquired outright prior to construction, by the construction of frontage/rearage roads or by a combination of both.

The ramp terminal is that portion adjacent to the through traveled way, including speed-change lanes, tapers, and islands. Ramp terminals may be the at-grade type, such as at the crossroad terminal of a diamond interchange, or the free-flow type where the ramp traffic merges with or diverges from through traffic at flat angles.

The begin and end control of access stations are noted on the construction plans.

Interchanges

The control of access for interchanges is measured from the ramp terminal to access location on the crossroad as shown in Figure 13-12. Determination of the length of need for control of access should be based on the following:acceptable level of service per Table 15-1 - Level of Service Guidelinesobserved and predicted crash rates

At existing interchanges, the control of access has generally been established and will likely vary in distance from Table 13-6. For each reconstruction project the existing control of access should be reevaluated to determine whether sufficient control of access exists. For projects where a new interchange is being constructed, control of access should be established based on Table 13-6.
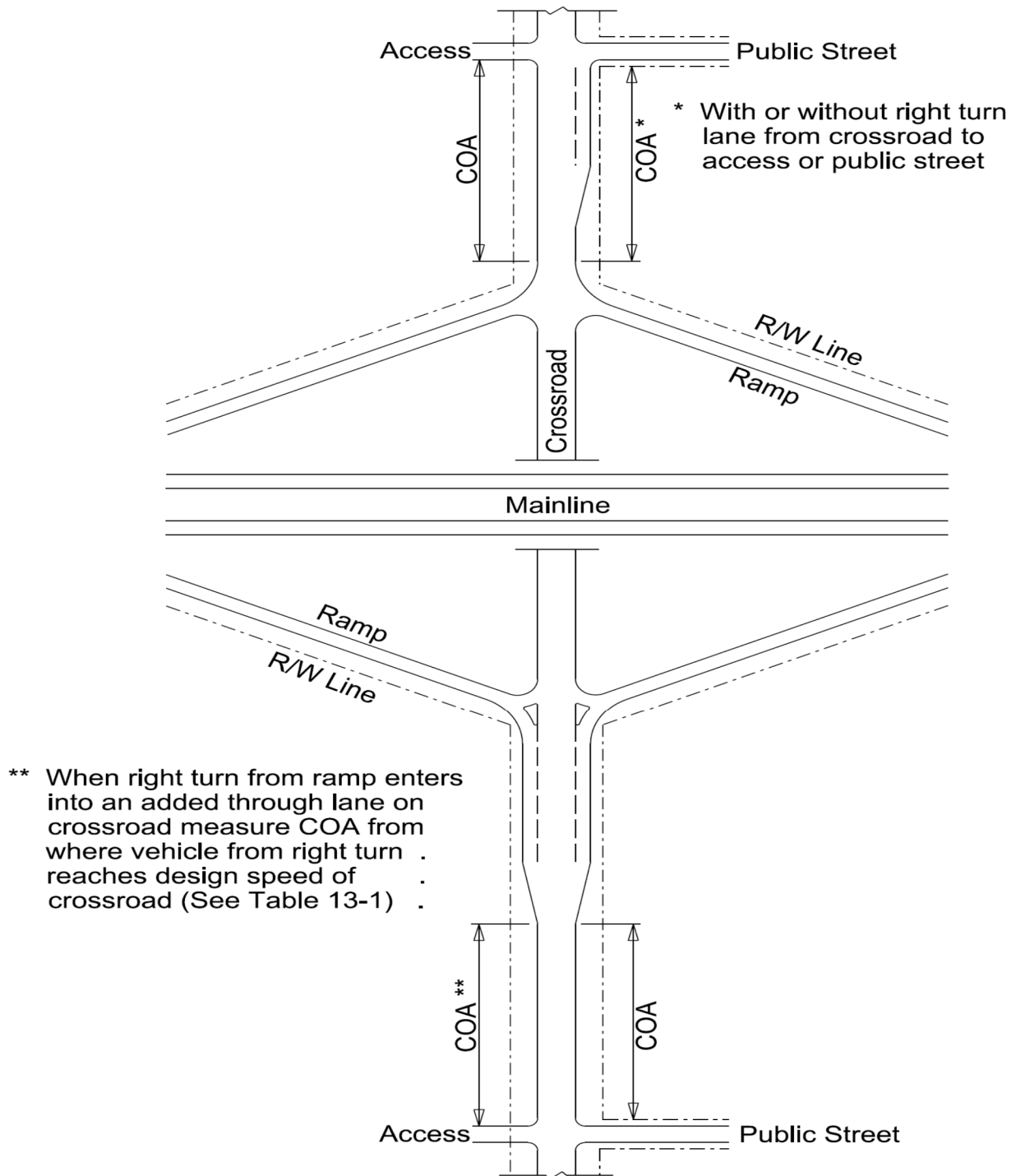
Access   Public Street

COA

COA*

* With or without right turn lane from crossroad to access or public street

R/W Line

Ramp

Crossroad

Mainline

Ramp

R/W Line

** When right turn from ramp enters into an added through lane on crossroad measure COA from where vehicle from right turn reaches design speed of crossroad (See Table 13-1)

COA**

COA

Access   Public Street

Figure 13-12   Control of Access at Interchanges

Grade Separated Locations

The control of access for grade separated locations is measured from the mainline right-of-way line to access location on the crossroad as shown in Figure 13-13.
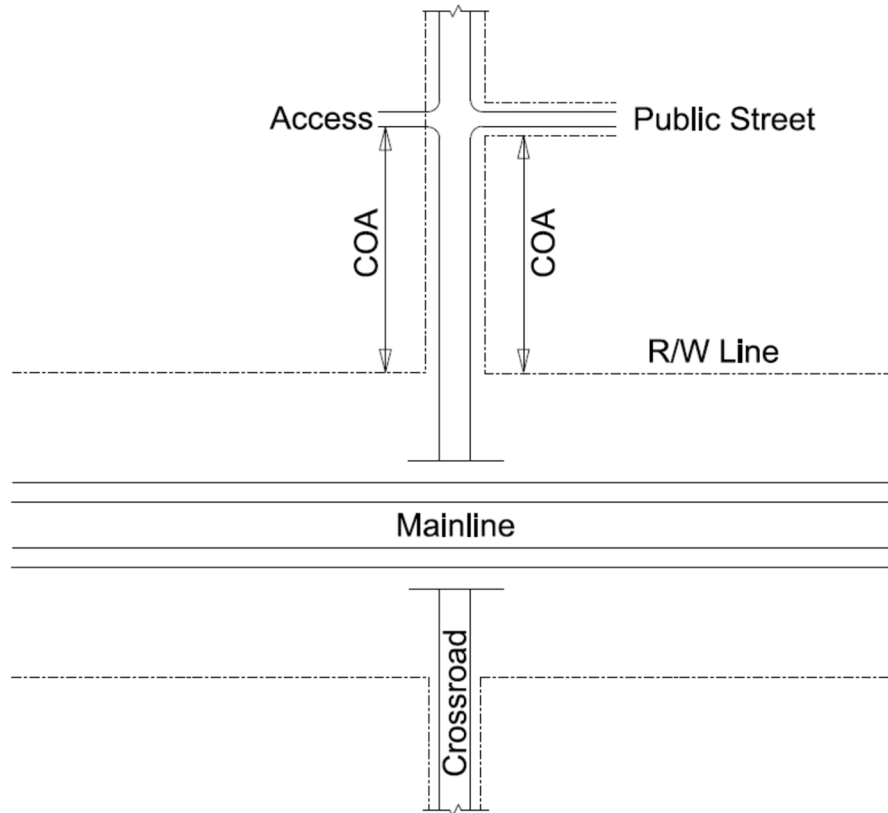

Figure 13-13 Control of Access at Grade Separated Locations

Table 13-6 Minimum Control of Access

Type of Improvement	Control of Access[1]	
	Urban	**Rural**
Reconstruction of Existing Interchange [2,3]	100'	300'
Construction of New Interchange [3]	660'	660'
Reconstruction and Construction of Grade Separated Crossing [3]	330'	330'

[1] If the minimum control of access distance may not be obtained because of existing development and potential of excessive right-of-way damages, a request for design variance may be submitted for review/approval from the FHWA Division Office.

[2] These distances are considered minimum per the current AASHTO publication *A Policy on Design Standards Interstate System*. Where possible more control of access should be obtained based on documented traffic or safety concerns.

[3] Where situations allow, consider extending the control of access to meet the access spacing shown in Figure 17-1 South Dakota Access Location Criteria.

Weaving Sections

Weaving sections are highway segments where the pattern of traffic entering and leaving at contiguous points of access results in vehicle paths crossing each other. Weaving sections may occur within an interchange, between entrance ramps followed by exit ramps of successive interchanges, and on segments of overlapping roadways.

Because considerable conflicts occur through weaving sections, interchange designs that eliminate weaving or remove it from the main facility are desirable. Weaving sections may be eliminated from the main facility by the selection of interchange forms that do not have weaving or by the incorporation of collector-distributor roads. Although interchanges that do not involve weaving operate better than those that do, interchanges with weaving areas are often less costly than those without.
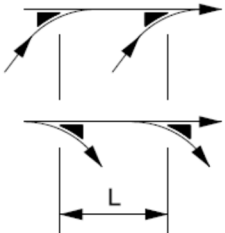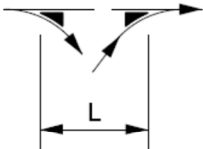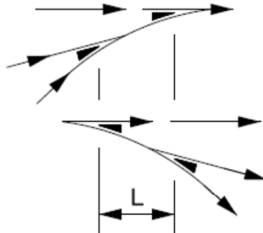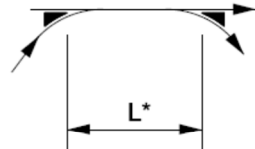
Designs to avoid weaving movements require a greater number of structures or larger and more complex structures, with some direct connections. Joint evaluation of the total interchange cost and the specific volumes to be handled is required to reach a sound decision between design alternates.

Where cloverleaf interchanges are used, consideration should be given to the inclusion of collector-distributor roads on the main facility, or possibly both facilities where warranted.

When weaving volume on a particular cloverleaf weave exceeds 1,000 vph, the level of service on the main facility deteriorates rapidly, thus generating a need to transfer the weaving section from the through lanes to a collector-distributor road.

Minimum weaving lengths, as shown in Figure 13-14, should be used on cloverleaf interchanges. These minimum weaving distances are not applicable to cloverleaf loop ramps, where the distance between gores is dependant on the ramp radii and the roadway and median widths.

The capacity of weaving sections may be seriously restricted unless adequate length and width are provided through the weaving section along with lane balance. Chapter 24 of the TRB Highway Capacity Manual addresses weaving area configurations and level of service calculations. Level of Service C is the preferred minimum level of service.

EN-EN OR EX-EX		EX-EN		TURNING ROADWAYS		EN-EX (WEAVING)			
						System to Service Interchange		Service to Service Interchange	
Full Freeway	CDR or FDR	Full Freeway	CDR or FDR	System Interchange	Service Interchange	Full Fwy.	CDR or FDR	Full Fwy.	CDR or FDR
MINIMUM LENGTHS MEASURED BETWEEN SUCCESSIVE RAMP TERMINALS									
1000 Ft	800 Ft	500 Ft	400 Ft	800 Ft	600 Ft	2000 Ft	1600 Ft	1600 Ft	1000 Ft

Notes:
FDR - Freeway Distributor Road
CDR - Collector Distributor Road
EN - Entrance
EX - Exit

The recommendations are based on operational experience and need for flexibility and adequate signing. They should be checked in accordance with the procedure outlined in the Highway Capacity Manual and the larger of the values is suggested for use. Also, a procedure for measuring the length of the weaving section is given in Chapter 24 of the Highway Capacity Manual. The "L" distances noted in the figures above are between like points, not necessarily "physical" gores. A minimum distance of 270 ft is recommended between the end of the taper for the first on ramp and the theoretical gore for the succeeding on ramp for the EN–EN (similar to EX-EN).

Figure 13-14 Recommended Minimum Ramp Terminal Spacing